Chronological Construction Sequence, Creep, Shrinkage, and Pushover Analysis of an Iconic 1960s Reinforced Concrete Building

O.A. Rosenboom, T.F. Paret and G.R. Searer
Wiss, Janney, Elstner Associates, Inc.

SUMMARY:
When analyzing a structure, the structure's "birth" is typically taken to be an instantaneous moment in time. The influence of the construction sequence and the time effects related to the material properties and structural behavior are typically and sometimes inappropriately ignored. Following the 2006 Kiholo Bay earthquake on the "Big Island" of Hawaii, a unique cracking pattern was observed in an iconic 1960s reinforced lightweight concrete building. In order to determine the cause of the cracking, a nonlinear finite-element model was developed using the Adina analysis program. In the analyses, the construction sequence and expected shrinkage were modelled. A nonlinear static pushover was also performed using the Capacity Spectrum Method. The analyses demonstrated that a state of sustained tensile stress (i.e. tensile creep) created from the construction sequence offered the best explanation for the unique cracking pattern.

Keywords: Tensile Creep, Shrinkage, Lightweight Concrete, Pushover Analysis, Capacity Spectrum Method

1. SIGNIFICANCE OF STUDY

Tensile creep in concrete is a phenomenon that has received relatively little attention by structural engineers because the tensile strength of concrete is not a codified component of the strength of reinforced concrete structures. We reinforce concrete precisely because it is weak in tension. It is expected to crack and its tensile strength is disregarded in design. As a result, when inclined cracks are observed in reinforced concrete shear walls after earthquakes, tensile creep is not the first mechanism that most engineers would begin to investigate. Most of us might begin with the assumption that the cracks were caused by the response of the structure to ground shaking. Tensile creep, however, can cause cracking in reinforced concrete structures, and certain circumstances increase the likelihood that it will cause cracking.

The structure described herein was especially vulnerable to the formation of inclined cracking from tensile creep because it was constructed with ultra-lightweight concrete and because its shear walls were supported in a manner that caused sustained inclined principal tensile stresses that approached the expected short-term tensile of concrete under the influence of gravity in the field of the walls. Due to its one-of-a-kind configuration, its seismic response is also less conceptually transparent than almost any other concrete shear wall structure. Segregating the influence of earthquake shaking from the influence of sustained loading was an immensely complex problem that required a multi-faceted seismological, engineering and materials-based investigation, including evaluation of floor response spectra and contents damage, with an unusually in-depth nonlinear time-dependent analysis phase.

2. BUILDING CHRONOLOGY

Rising in 1965 from a secluded beach on the Kohala coast of Hawaii County, an iconic reinforced concrete building was designed to conform to its natural surroundings. On a sloping site, hotel room floors are perched high atop massive cruciform-shaped piers, allowing sweeping views of the ocean throughout the main floor of the hotel and sea breezes to flow to the ocean side and mountainside towers through an open atrium that bisects the structure longitudinally (Figure 1). As a consequence of the hotel's stepped floor plans, the reinforced concrete shear walls that support the gravity loads for each floor are staggered at each level, allowing sunny lanais but creating a complex load path for both gravity and lateral loads. The building has major vertical discontinuities at every level, including the primary discontinuity above the main level where all the loads originating in the three elevated floors of hotel rooms (denoted as the 5th through 7th floors) are transferred by widely-spaced transfer girders and a thickened slab into massive cruciform piers, with the shear walls spanning between the transfer girders. The transfer of all gravity loads from the shear walls into transfer girders at the transfer level requires that the shear walls support gravity loads as wall-beams rather than as compression members, which subjects them to a sustained shear/principal tension field. Lateral loads are resisted by lightweight reinforced concrete cruciform piers below the transfer level and by shear walls above. Due to the sloping site, the cruciform piers on the ocean side tower are significantly taller than the mountainside tower (approximately 31 feet compared with 12 feet), further complicating the lateral load path. The floors at and above the transfer level have significantly fewer shear walls in the direction parallel to the shoreline, in which direction lateral loads are also resisted by slab-wall frames. There are six "towers" total, arranged into three wings: north, center, and south, each of which is unique and behaves differently under lateral loads due to the presence or lack of interconnecting diaphragms, pedestrian bridges, and elevator shafts. The north, center, and south wings are seismically separated by nominal gaps of about two inches.

Figure 1. Original architectural model (before addition of penthouse)

Due to the high cost of shipping construction materials to what was in the 1960s a very remote corner of the Big Island, the fine and coarse aggregate for the concrete was sourced locally from the Volcanite, Ltd. Puuwaawaa quarry located at a cinder cone on the slopes of the Mauna Loa volcano (Bureau of Mines 1968). The aggregate is predominantly vesicular, extremely lightweight, floats in water, and is easily pulverized. The specified concrete strength on the original construction drawings was 20.7 MPa and the specified mix resulted in a dry density of approximately 16.5 kN/m^3.

By happenstance, one of the original California-based structural designers of the hotel is a retired Principal in the authors' firm who was able to provide invaluable information regarding the original design and construction, including that the seismic design of the building was based on the 1961 UBC. except that it was assumed that the building was situated in Zone 3, similar to California at the time. In addition to being involved in the design of the building, he also visited the site multiple times during construction and took hundreds of photographs. The photographs document the reinforcing, the shoring configurations and staging, as well as the construction sequence -- when each cruciform pier, transfer girder, beam, wall and slab was poured with respect to surrounding elements.

In 1972, a lightweight steel framed "penthouse" that extended the full length of the entire structure (the "8th" floor) was added. Framed into and between the existing elevator shafts, the roof of the penthouse was continuous between wings and straddled the seismic gaps below without having any seismic joints built into it. Thus, relative movement of the concrete "towers" below transmitted forces into the penthouse framing. Lateral loads originating in the penthouse were resisted by the existing concrete elevator shafts in the longitudinal direction and by tension-only steel rods in the transverse direction. In 1997, a full renovation of the hotel was completed with notes on the drawings mentioning repair (as required) of all concrete surfaces.

3. SEISMOLOGICAL BACKGROUND

The ground shaking hazard in Hawaii is among the highest in the United States, yet the mechanisms and attenuation for Hawaii-style earthquakes are not well understood (EERI 2006). The seismology of Hawaii County is typified by shallow décollement earthquakes occurring at depths of 5 to 15 kilometers (km) or deeper mantle earthquakes occurring at depths of 30 to 40 km. Numerous large and small earthquakes shook the building during the first two decades after its construction, followed by a two-decade period of quiescence. Several large earthquakes have occurred since construction of the subject building, including the 1973 Mw 6.2 Honomu earthquake, the 1975 Mw 7.2 Kalapana earthquake and the 1983 Mw 6.7 Kaoiki earthquake. Certain smaller earthquakes during this period, including one in 1979 and one in 1982, are especially notable due to their proximity to the building: each was approximately 4 km in distance from the subject building to the epicenter. Relationships have been put forth for the attenuation for shallow décollement earthquakes (Munson and Thurber 1997) but the attenuation relationship of deeper mantle earthquakes in this area is unknown.

On October 15, 2006, a Mw 6.7 earthquake was recorded at a depth of 38 km, occurring 18 km from the subject building. Seven minutes later, a Mw 6.0 earthquake was recorded at a depth of 18 km, occurring 21 km from the subject building. The depth and location of the first earthquake would indicate a mantle earthquake, but the depth of the second earthquake and the aftershock sequence would suggest they are from separate seismic sources. The atypical high frequency content in the recorded motions of the 2006 earthquake cannot be explained with existing California or décollement attenuation relationships (EERI 2006).

Others have created isoseismal maps of certain historical earthquakes and have found that the shaking intensity, i.e. the peak ground accelerations (PGA), in Hawaii must be larger than on the mainland to result in the same Modified Mercalli Intensity (MMI) (Wyss and Koyanagi 1992). Values of MMI of the historical earthquakes are provided in Table 1 at the subject building along with 2006 intensities from USGS ShakeMaps. For technical reasons that cannot be discussed here due to space limitations, the authors do not believe that the ShakeMap data generated during the 2006 events provides reliable information.

Table 1. Significant earthquakes at the subject building (Intensity from Wyss and Koyanagi 1992 or USGS)

Year	Event name	Mw	Depth (km)	Distance (km)	MM Intensity
1973	Honomu	6.2	50	76	VI
1975	Kalapana	7.2	5	111	V
1979	[unknown]	5.1	11	4	[unknown]
1982	[unknown]	4.8	19	4	[unknown]
1983	Kaoiki	6.7	12	74	V
2006	Kiholo Bay	6.7	38	18	VII*
2006	Mahukona	6.0	18	21	VI*

* From USGS ShakeMaps

The sparse network of strong motion instruments in Hawaii County resulted in a dearth of recordings of the 2006 earthquakes. The closest station to the hotel, at a distance of 11km from the building, was an instrument at the Waikoloa Hotel in Anaehoomalu which registered a PGA of 0.187 g and is situated on a rock site similar to the subject building. Several other stations recorded much higher PGA values but are located in areas of stiff soils, or Site Class D, or situated in areas of volcanic ash deposits known to exhibit high amplification (EERI 2006).

4. EARTHQUAKE DAMAGE ASSESSMENT

The hotel sustained damage in the 2006 Kiholo Bay earthquakes in several vulnerable locations. A poorly anchored 5-story tall concrete exhaust flume, added as part of the 1972 addition, partially disconnected from an elevator shaft. The 1972 8th floor addition, whose roof straddled the seismic separations between the wings, sustained severe non-structural and moderate to severe structural damage: several of its tension-only cables fractured or buckled and many of its framing connections to the elevator shafts, later identified as having been damaged from prior earthquakes and then repaired, were also damaged.

After the earthquake, distinctive patterns of inclined cracking were noted in most of the painted shear walls, and assessment of the condition of the building was initiated by multiple parties. The authors' investigation, seeking to understand the cause and significance of the cracking, approached the subject from a variety of perspectives, but detailed physical examination of the cracking immediately after the earthquake was not possible due to the owner's decision to immediately grind finishes from the surface of the walls along every visible crack. As crack maps along typical reinforced concrete wall lines were prepared and conceptually evaluated, however, patterns that were inconsistent with the stress fields normally associated with seismic ground shaking began to emerge. For example, "identical" wall segments on opposite sides of the central atrium exhibited mirror-image inclined crack patterns and some of the inclined cracking, particularly on discontinuous cantilevered wall segments, seemed much more likely to be associated with gravity stress fields rather than that induced by an earthquake (Figure 2). In any case, in the relatively few areas in which existing wall surfaces were not removed, many of the cracks exhibited surface repairs, indicating that even cracks that might be associated with seismic ground shaking pre-dated the 2006 event. These cracks were later demonstrated through testing to be deeply carbonated.

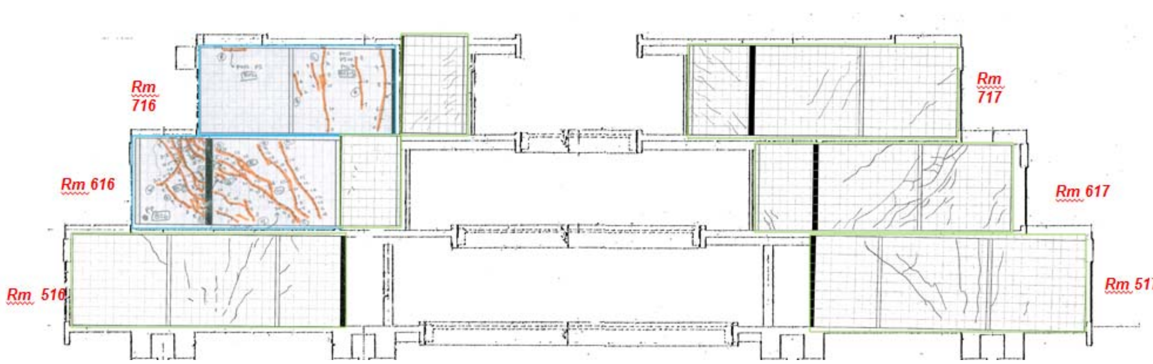

Figure 2. Crack map of typical wall line on ocean side (left) and mountainside (right) towers.

5. MATERIAL TESTING

The specifics of the cracking observed in the hotel, especially that the concrete in the building was extraordinarily lightweight and that the crack patterns, though inclined, exhibited certain non-seismic characteristics, indicated that another phenomenon was possibly at work. A detailed study of the physical properties of the concrete was embarked on as an extension of the post-earthquake damage assessment. Primary goals for the materials study were to assess shrinkage characteristics, compressive strength, and tensile strength under sustained loading.

In the 1970s, the University of Hawaii (UH) studied material properties of concrete with a variety of Hawaiian aggregates; one of the aggregates they tested was Volcanite, from the same quarry which had sourced the aggregate for the subject building (Hamada *et al.* 1972). While normal concrete typically has an ultimate shrinkage strain of around 500 $\mu\varepsilon$ (microstrain), UH reported one-year tested values for the shrinkage of Volcanite around 1000 $\mu\varepsilon$.

The materials study, intended to serve as a base for re-creation of the concrete, included coring of the existing walls to obtain samples of the extant concrete for compression strength testing and for petrographic examination. Via the petrographic studies, estimates of the original water-to-cement (w/c) ratio were made and the mix design for the original concrete was reverse-engineered. Coarse and fine volcanic aggregate samples from the original Volcanite quarry were obtained, compared to those found in the concrete samples from the shear walls, and found to be an exact match. After gaining permission from the guardian of the now-abandoned quarry to collect larger quantities of aggregate, batches of the concrete were prepared in the laboratory and cylinders for compression testing were prepared. A test procedure for direct tensile testing of concrete "bobbins" was also developed and an apparatus for subjecting the spindles for long-term tensile testing was fabricated. A set of bobbins was fabricated, with a subset of these tested to failure under short term tensile loading and statistically analyzed. Short-term tensile strengths for 10 test bobbins averaged 2.2 MPa with a coefficient of variation of 11 percent. These tensile strength values were employed to set the range of stress to which the remaining bobbins were subjected to assess their long-term response to tensile loading. These were subjected to constant stress intensities between a minimum of 40 percent and a maximum of 90 percent of the mean tested short-term strength. These tests are still in progress and will eventually be reported elsewhere, but after 2-1/2 years, all the bobbins with long-term tensile stress exceeding 70% of the mean short-term strength have broken, as well as one as low as 60%.

6. STRUCTURAL MODELING

Due to the highly complex load paths present throughout the building for both gravity and lateral loads, the complex construction sequence indicated by construction era records, the unusual properties of the concrete, and the difficulty of relating much of the existing cracking to either gravity or lateral forces unambiguously, the authors embarked on a series of detailed structural analyses designed to clarify the cause of the observed cracking. Almost from the outset of the analysis phase of the project, it became clear that simple linear-elastic substructure models of the building, loaded with gravity and subjected to either linear-static or response spectrum loads generated from the ground motion recorded in the Waikoloa Hotel, could not explain the primary cracking patterns observed in the building. Neither the cracking in the cantilevered wall segments nor the mirrored cracking appeared to be explained by seismic response. Indeed, when these simple models and hand calculations were subjected to earthquake motions much greater than the 2006 record in order to highlight areas of structural vulnerability, the bulk of the predicted damage occurred in the cruciform piers that supported the room levels of the hotel, a location in the real building that decidedly lacked significant cracking. The convoluted gravity and lateral load path of the building would ultimately require two distinct additional analytical efforts to explain the cracking. The first consisted of linear models of all portions of the buildings to ascertain the effects of load path irregularities associated with vertical discontinuities, the cruciform piers and the diaphragms. The second consisted of a chronological "pushover" of the building from the time of concrete casting, taking into account materials data from our testing program and construction sequencing.

The base analysis for the building consisted of a three-dimensional linear model constructed mainly of shell elements in SAP 2000 Version 14 (Computers and Structures 2009). The entire south wing of the building including the upper hotel portion, penthouse, and all of the structural elements in the back-of-house service areas, tunnels, and commercial spaces were modelled. A detailed discussion of the linear models is not possible due to space limitations. One of the complexities in the structure included that many of the cruciform piers, particularly those supporting the mountain side "tower", are integrated with foundation walls, and other piers, particularly those supporting the beach side "tower" are integrated with roof diaphragms of seismically separate low-rise concrete out-buildings located below the transfer level (Figure 3). Several pedestrian bridges connecting the mountain side and ocean side diaphragms were constructed with the ability to slip in only one direction, and were modelled as compression-only struts. Comparing the lack of consistency between the damage predicted by the linear model with observations after the 2006 earthquake demonstrated that the earthquake loading spectra used in the analysis were in fact substantially larger than the ground motions experienced at the building and resulted in predicted damage in elements that were essentially undamaged. Sub-studies conducted with the linear model also informed the selection of representative substructures from the building for the follow-up nonlinear studies conducted in ADINA and described below (Adina Research and Development 2009).

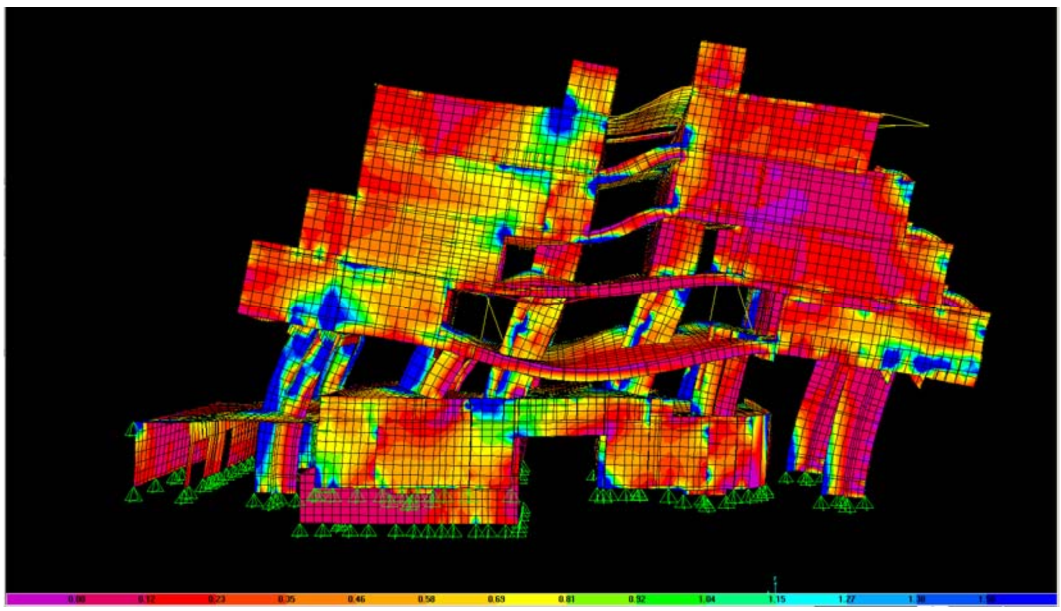

Figure 3. Principal tensile stresses in linear model of south wing under amplified seismic loading

Three-dimensional (3-d) solids were chosen as the primary modelling element in the ADINA studies to avoid the double-counting of structure throughout the transfer level that would have occurred if shells were used. The smeared cracking concrete model in ADINA allows the modelling of embedded steel reinforcing discretely modelled as truss bars with constraints provided at intersecting 3-d solids. This modelling technique allows consideration of each piece of reinforcing steel in the building (other than those such as hooks and splices that relate to bond or development) but assumes perfect bond between the reinforcing and concrete because ADINA is not yet able to model the bond-slip characteristics of the reinforcing steel. A representative three-wall four-pier building substructure that takes into consideration the repetitive nature of the stacked shear walls was selected for detailed nonlinear analysis and was modelled with symmetric boundary conditions which included "half-cruciform" piers (Figure 5). Two general models were created, one with tall cruciform piers representing a condition on the ocean side, and one with shorter piers representing the mountain side condition. Although all the stacked transverse shear walls in the building are discontinuous, the three-wall model captures the behavior of the two distinct types of wall stacks that exist in the building -- those that are supported directly on cruciform piers and those that are supported on transfer girders. All of the longitudinal shear walls in the building are also discontinuous but will not be discussed here due to space limitations.

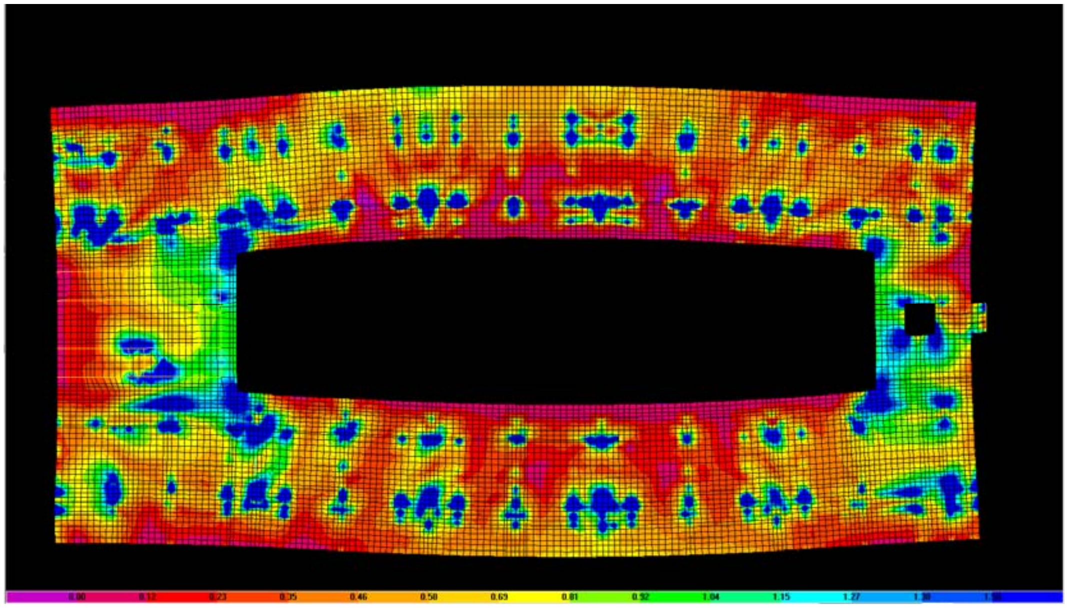

Figure 4. Principal tensile stresses at 5th floor under amplified seismic loading towards bottom of page

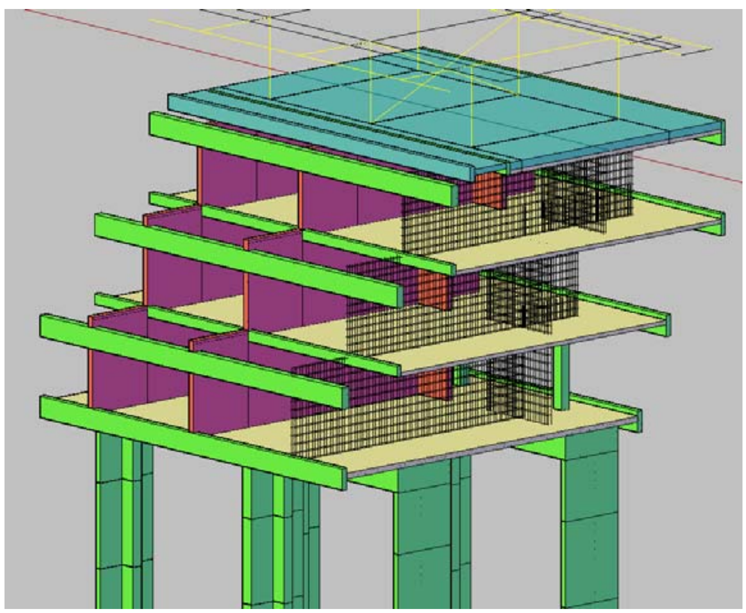

Figure 5. Schematic of ADINA model with some removed 3-d solids to reveal embedded steel

The transfer framing at the 5th floor is significant to the behavior of the building, particularly in that it introduces different support conditions for adjacent shear walls. Although transfer girders span continuously in the longitudinal direction along each line of cruciform piers and support the "end" of each shear wall, no girders support the length of any shear wall; seismic loads from the shear walls are therefore transmitted directly through the slab, and the shear walls are "self-supporting" beams that span between the transfer girders. The transfer level slab is slightly thicker than the typical slab on upper floors (8 inches compared with 6 inches). To model symmetry for the 3-d solid slab elements, constrained end plates (modelled as shell elements) were employed to prevent the slab from rotating out-of-plane of the loading.

The ADINA analyses were employed to introduce construction sequence and time-dependent material properties into the study. Like an erector set, each structural component in the ADINA model -- including the 3-d solids and the reinforcing truss bars -- was "born" at a discrete time, introduced first as dead load to represent wet concrete as it was cast, then as a structural material with stiffness. The sequence of the "births" and the location of the "construction joints" were determined from the hundreds of construction-era photographs available to the authors. Shoring configurations and staging as represented in the photographs were also considered in the model construction, going so far as to consider the effects of inserting and removing individual "shoring" frame elements to support the 7th floor cantilever slab and beam system prior to the "casting" of the 7th floor walls. Besides having different concrete strengths between the various structural components such as transfer girders, beams, columns, and walls (as specified on the original construction drawings), the concrete material model that was used in the analysis was time-dependent. The model considered such effects as tensile creep, compressive creep, and shrinkage, and also differentiated between short- and long-term material properties. For example, shrinkage strains were ramp-applied to each concrete "pour" immediately after its "birth", thereby capturing the restraint induced by supporting or surrounding elements. The tensile strength of the concrete was modelled as 1.4 MPa under long-term loading (such as sustained dead load) but modelled as 2.1 MPa under short-term loading regimens (such as seismic loading). Other parameters in the concrete model such as the fracture energy and the post-cracking transfer of stresses across the crack were also modelled as time-dependent. The embedded reinforcing steel and the structural framing in the penthouse were modelled as elastic-plastic.

The gravity stress field combined with the ramp-applied shrinkage regimen to each structural component after birth, but absent applied lateral forces, revealed the patterns of tensile stress vectors shown in Figure 6. The addition of each stacked, offset floor altered the horizontal center-of-mass of the building, shifting it towards the courtyard with each floor addition. The final shift occurred in 1972 when the 8th floor penthouse was constructed atop discontinuous walls (Figure 7). The pattern of cracking in the walls revealed in the gravity plus shrinkage analysis was essentially identical in every wall, regardless of whether the stacked walls are supported by tall cruciform piers (ocean side), short cruciform piers (mountain side), or are located between the pier and are supported by transfer girders. This nearly invariant crack pattern from dead load replicated observations in building after the 2006 earthquake.

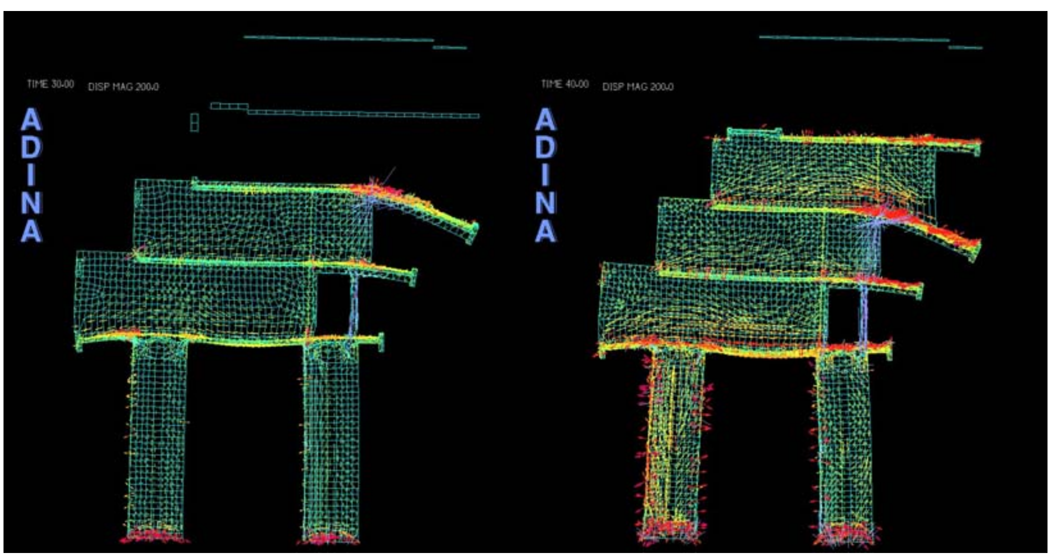

Figure 6. Principal tensile vector plot. Construction sequence to 6th floor (left) and to the 7th floor (right)

These findings were consistent with the results of parallel studies conducted to explore the effects of the earthquake, including scanning electron microscopic (SEM) studies of paint and plaster and evaluation contents damage based on fragility analyses. The SEM studies revealed that the typical walls had been re-painted about 10 times during the forty years between original construction and the 2006 events. The first five layers were extraordinarily thick, with up to one gallon of paint being used for every 100 square feet of wall surface, while the second five layers indicated more normal coverage rates. This painting "sequence" is consistent with the postulated scenario that the building, which first experienced extensive cracking in its walls during and after construction, was then subjected to a series of earthquake events during its first twenty years, followed by a twenty year period of earthquake quiescence. Each time the cracks "jiggled" during an earthquake event, heavy paint application was required to cover the cracks. When the 2006 events occurred, these cracks once again propagated through the paint.

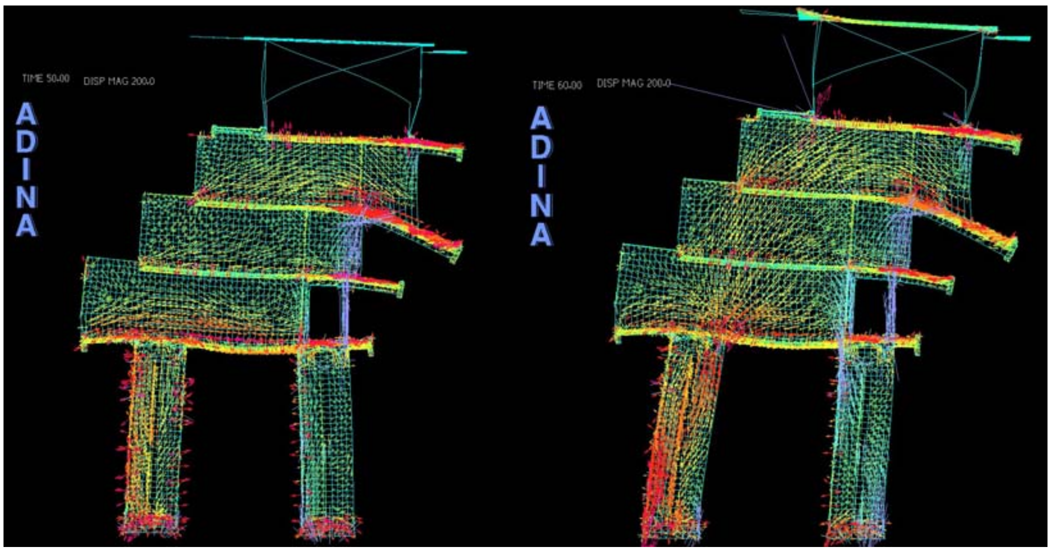

Figure 7. Principal tensile vector plot. Construction sequence to 8th floor (left) and after applying seismic pushover loads calibrated to the recorded accelerations at the Waikoloa Hotel (right)

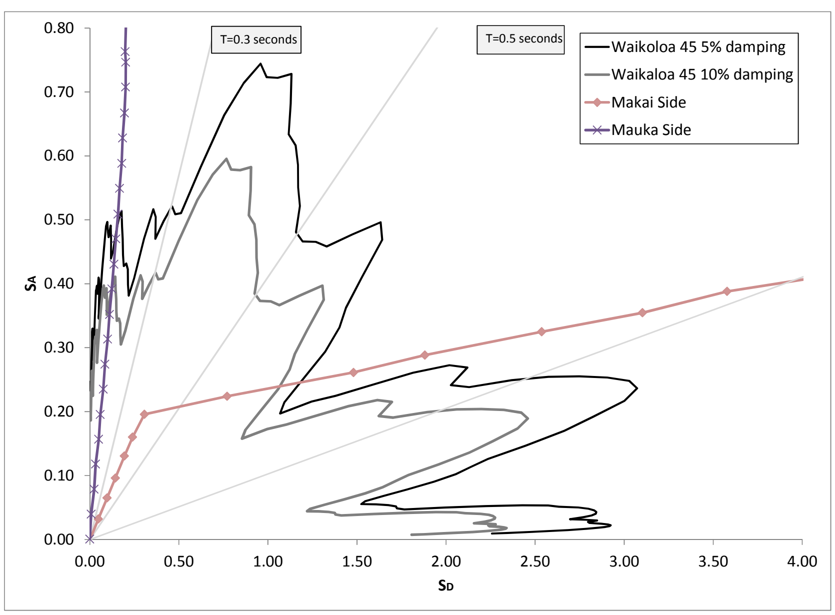

Figure 8. ADRS pushover for mountain side (mauka) and ocean side (makai) submodels

Also of note, because the property was a hotel, a detailed inventory of all contents damage was kept, including damage to furniture, televisions, glassware, desk sets, etc. These inventories were maintained on a floor-by-floor basis, enabling calculation of damage rates and direct comparison of the calculated rates to published fragilities and to floor response accelerations predicted by the structural analyses. The comparisons demonstrated that the response accelerations must have been substantially less than those predicated on the Waikaloa Hotel records, which also helped to explain why the predicted yielding in the cruciform piers did not occur.

7. CONCLUSIONS

Expanding a non-linear static analysis to include time-dependent material properties, construction sequence, and the effect of creep and shrinkage reveals the importance of these effects on structural behavior, particularly when complex gravity load paths and restraint conditions create non-intuitive stress fields. For the subject building, a historic reinforced concrete shear wall building with unique lightweight aggregates, the chronological pushover confirmed field observations that cracking due to shrinkage and tensile creep from dead load and construction sequencing was more predominant in the building than any damage related to the 2006 earthquakes.

ACKNOWLEDGEMENT

Sven Thomasen, structural engineer at WJE San Francisco.

REFERENCES

ADINA Research and Development (2009). ADINA Theory and Modeling Guide, ARD09-7.

American Society for Civil Engineers (2006). ASCE 41: Seismic Rehabilitation of Existing Buildings, ASCE, Reston, VA.

Bureau of Mines (1968). Minerals Yearbook Area Reports: Domestic 1964, US Government Printing Office.

Computers and Structures, Inc. (2009). CSI Analysis Reference Manual for SAP2000, ETABS, SAFE and CSiBridge, Berkeley, California.

Earthquake Engineering Research Institute (2006). Compilation of Observations of the October 15, 2006 Kiholo Bay (Mw=6.7) and Mahukona (Mw=6.0) Earthquakes, Hawai'i, Oakland, CA.

Federal Emergency Management Agency. (1998). FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual, FEMA, Washington DC.

Federal Emergency Management Agency. (1998). FEMA 306: Evaluation of Earthquke Damaged Concrete and Masonry Wall Buildings: Technical Resources, FEMA, Washington DC.

Hamada, H.S., Watari, J.T., Chiu A.N.L. (1972). Creep and Shrinkage of Concrete made from Hawaiian Aggregates. *University of Hawaii Department of Civil Engineering Technical Report CE71-R2.*

Mahaney, J.A. Freeman, S.A., Paret, T.F., Kehoe, B.E. (1993). The capacity spectrum method for evaluating structural response during the Loma Prieta earthquake, *Proceedings 1993 National Earthquake Conf., Memphis*, 501-510.

Munson, C.G., and Thurber, C.H. (1997). Analysis of the attenuation of strong ground motion on the island of Hawaii. *Bulletin of the Seismological Society of America*. Volume 87: 4, 945-960.

Wyss, M. and Koyanagi, R.Y. (1992) Isoseismal Maps, Macroseismic Epicenters and Estimated Magnitudes of Historic Earthquakes in the Hawaiian Islands, *U.S.G.S. Bulletin.* 2006: 93.